CHAPTER 2

GENERAL DESIGN CONSIDERATIONS

Section I. Types of Retaining Walls

2-1. Common Types of Retaining Walls. The most common types of retaining walls are gravity concrete, cantilever T-type reinforced concrete, and cantilever and anchored sheet pile walls. Gravity and cantilever reinforced concrete walls are covered in this manual and illustrated in Figure 2-1. Alternate types of retaining walls, including mechanically stabilized backfill and precast modular gravity walls, are covered in Chapter 10. An example of one type of alternate retaining wall is shown in Figure 2-1. Counterfort and buttressed reinforced concrete walls are less commonly used and are not specifically discussed in this manual. Much of the conceptual information and the information in Chapters 3 and 9 is applicable to all types of walls.

2-2. Gravity Concrete Wall. A gravity wall (Figure 2-1) consists of mass concrete, generally without reinforcement. It is proportioned so that the resultant of the forces acting on any internal plane through the wall falls within, or close to, the kern of the section. A small tensile stress capacity is permissible for localized stresses due to extreme and temporary loading conditions.

2-3. Cantilever Reinforced Concrete Wall. A cantilever T-type reinforced concrete wall (Figure 2-1) consists of a concrete stem and base slab which form an inverted T. The structural members are fully reinforced to resist applied moments and shears. The base is made as narrow as practicable, but must be wide enough to ensure that the wall does not slide, overturn, settle excessively, or exceed the bearing capacity of the foundation. The bottom of the base should be below the zone subject to freezing and thawing or other seasonal volume changes. The T-type wall is usually the most economical type of conventional wall and is more widely used than any other type for common retaining wall heights.

2-4. Alternate Types of Retaining Walls. Retaining walls using mechanically stabilized backfill (Figure 2-1) and precast modular gravity walls can be substantially more economical to construct than conventional walls (Leary and Klinedinst 1984). However, a short life, serious consequences of failure, or high repair or replacement costs could offset a lower first cost. In addition, the design engineer must assure the overall adequacy of the design since the manufacturer of the wall may provide only that part of the design above the foundation. Chapter 10 covers mechanically stabilized backfill systems and precast modular gravity walls.

Section II. Types of Flood Walls

2-5. Common Types of Flood Walls. The most common types of flood walls are cantilever T-type and cantilever I-type walls. Examples of these walls are shown in Figure 2-2.

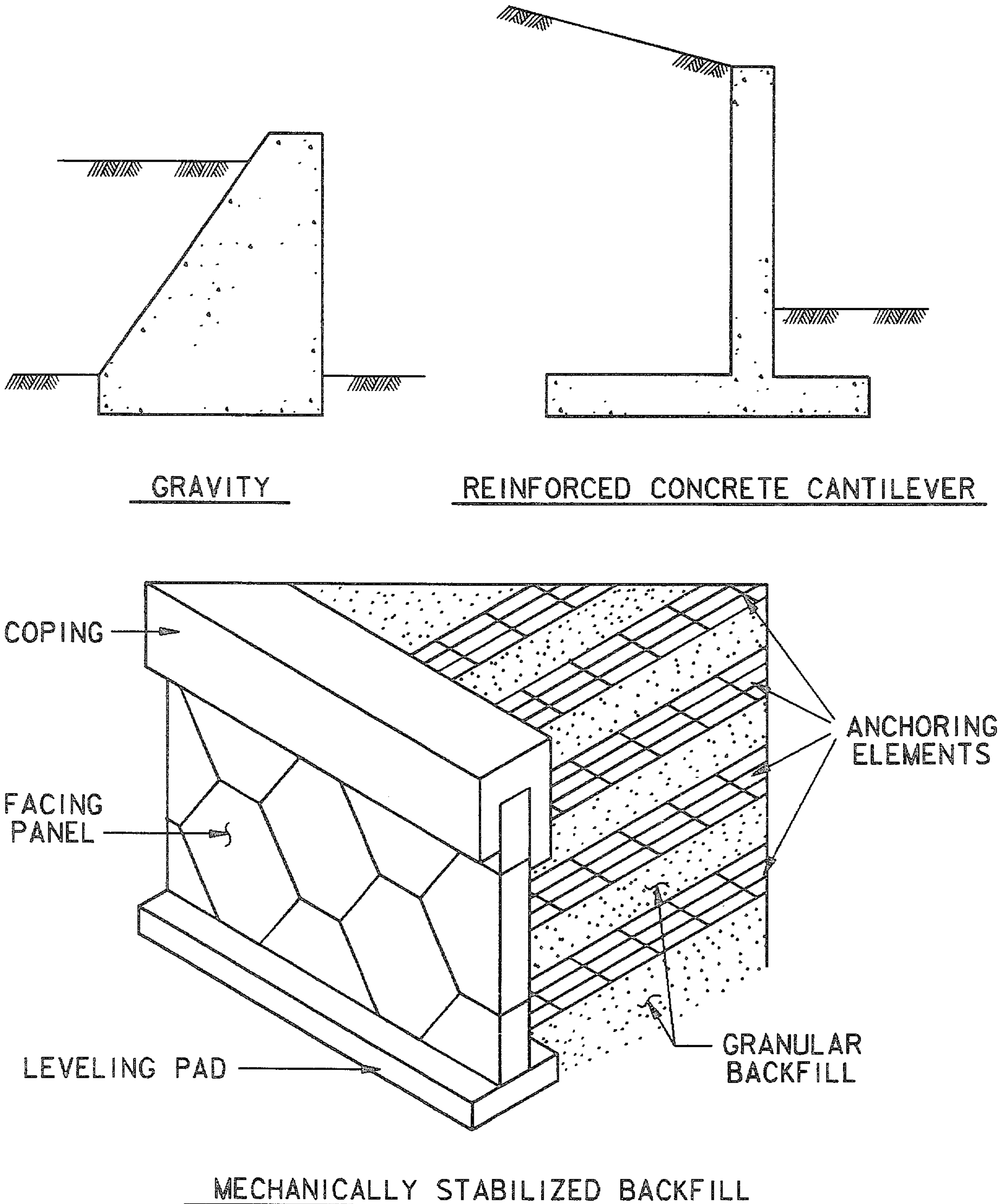


Figure 2-1. Types of retaining walls

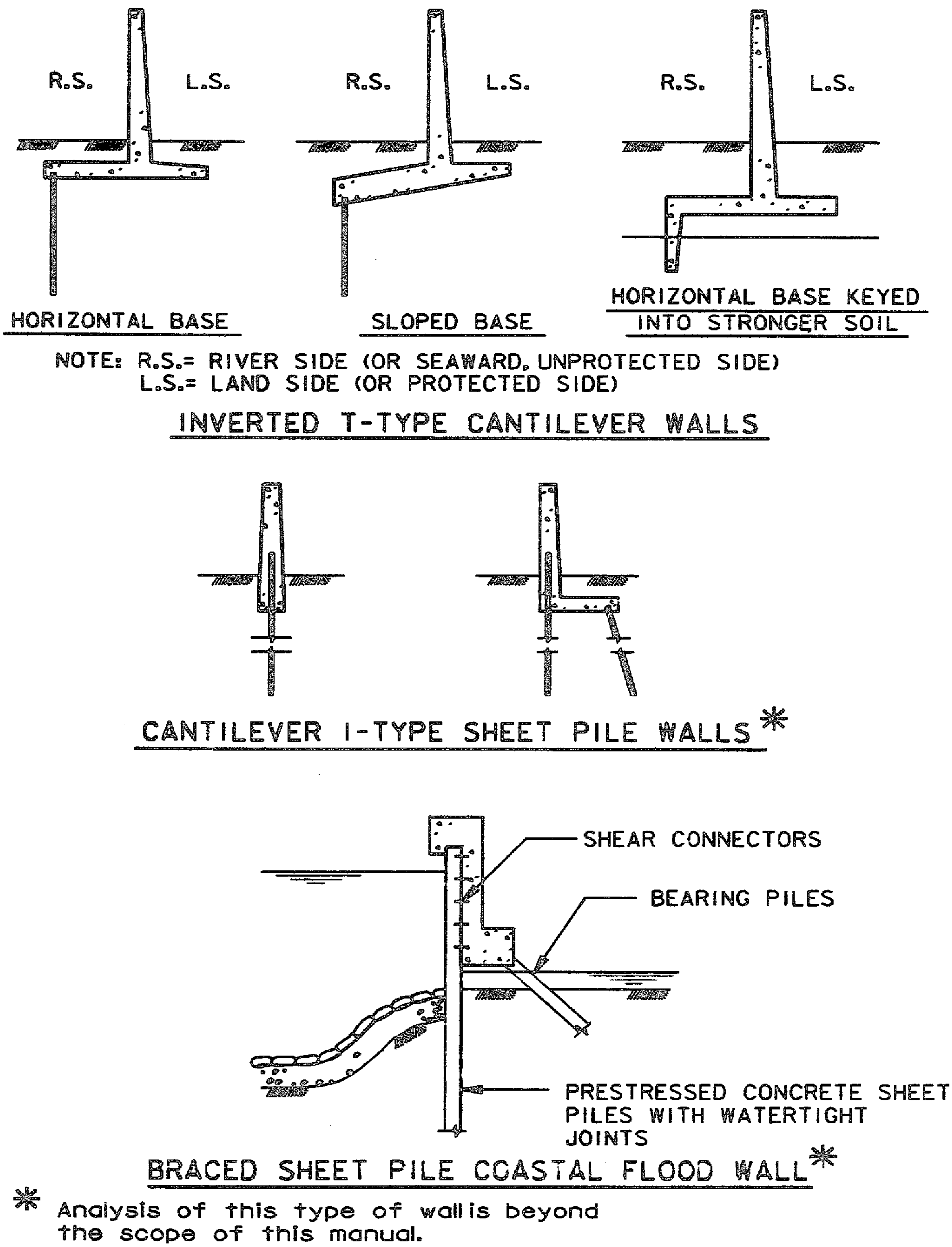


Figure 2-2. Types of flood walls

2-6. Cantilever T-Type Wall. Most flood walls are of the inverted T-type (Figure 2-2). These walls are discussed in detail in Chapter 7. The cross bar of the T serves as a base and the stem serves as the water barrier. When founded on earth, a vertical base key is sometimes used to increase resistance to horizontal movement. If the wall is founded on rock, a key is usually not provided. Where required, the wall can be supported on piles. A sheet pile cutoff can be included to control underseepage or provide scour protection for the foundation. T-type walls may be provided with a horizontal or sloped base. The advantages of sloped and horizontal bases are discussed in paragraph 7-5.

2-7. Cantilever I-Type Wall. I-type flood walls consist of driven sheet piles capped by a concrete wall (Figure 2-2). I-walls are most often used in connection with levee and T-wall junctions or for protection in narrow restricted areas where the wall height is not over 8 to 10 feet, depending on soil properties and geometry. The design of these types of walls is beyond the scope of this manual.

2-8. Other Types of Flood Walls.

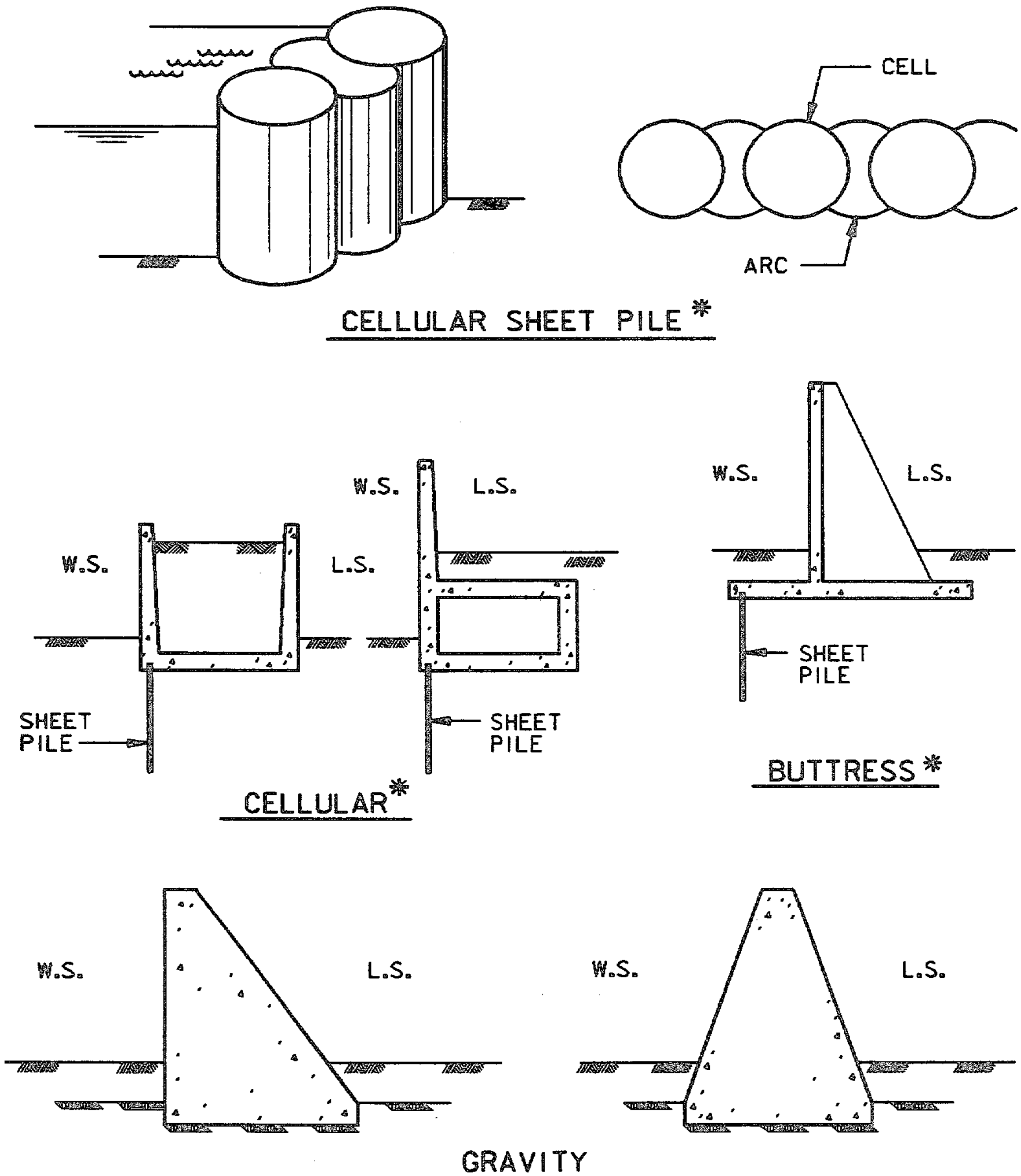
a. Braced Sheet Pile Flood Wall. This wall consists of a row of vertical prestressed concrete sheet piles, backed by batter piles connected to the sheet piles by a cast-in-place horizontal concrete beam with shear connectors as required to resist the vertical component of load in the batter pile (Figure 2-2). This type of wall has been used for coastal flood walls. It is ideal for wet areas because no excavation or dewatering is required to construct the wall. The disadvantage is that it is more indeterminate than other wall types. The design of this wall is beyond the scope of this manual.

b. Less Commonly Used Types. There are various other types of walls that may be used for flood walls such as: buttress, counterfort, gravity, cellular, and cellular sheet pile, some of which are shown in Figure 2-3. These walls, except for the gravity wall, are beyond the scope of this manual.

Section III. Differences Between Retaining and Flood Walls

2-9. Purpose of Walls. A retaining wall is any wall that retains material to maintain a change in elevation whereas the principal function of a flood wall is to prevent flooding (inundation) of adjacent land. A floodwall is subject to water force on one side which is usually greater than any resisting earth force on the opposite side. A wall may be a retaining wall for one loading condition and a flood wall for another loading condition. The flood loading (surge tide, river flood, etc.) may be from the same or the opposite direction as the higher earth elevation.

2-10. Seepage and Leakage Control Requirements. All water-retaining structures may be subject to seepage through, under, and around them. Inadequate control of seepage may affect the stability of a flood wall regarding uplift or loss of support resulting from erosion. Properly controlled seepage, even if quantities of flow remain large, presents little or no hazard. Control of



NOTES: W.S.= WATER SIDE (OR SEAWARD, UNPROTECTED SIDE)
L.S.= LAND SIDE (OR PROTECTED SIDE)

* Analysis of this type wall is beyond the scope of this manual.

Figure 2-3. Less commonly used flood wall types through-seepage is provided by water stops. Retaining walls rarely need seepage protection other than to relieve the hydrostatic load on the fill side of the wall. Water stops are used in retaining walls to prevent water passage from the backfill through the vertical joints. Seepage control and water stops are more fully discussed in paragraphs 3-23, 6-4e, 6-6, 7-4, and 7-13.

2-11. Wall Stability. Generally, it is more difficult to design stable flood walls than retaining walls. By their very nature, flood walls are usually built in a flood plain which may have poor foundation conditions. Uplift is always a critical item with flood walls but seldom a problem with retaining walls since the loads acting on a retaining wall are usually soil backfills. The water load on a flood wall can be more severe, especially when wave loadings are applicable. When the ground-water surface is near or above the wall footing, a common occurrence with flood walls, the allowable bearing capacity of the soil is reduced. The reduction of stability, due to the erosion of the earth cover over and beyond the base, must be considered.

2-12. Special Flood Wall Monoliths. Careful attention must be given to wall monoliths that have loading, support, or other conditions that vary along the length of the monolith. These monoliths, which may include closure structures, pipeline crossings, corner structures, etc., must be analyzed as complete three-dimensional entities instead of the usual two-dimensional unit slices.

2-13. Design Philosophy. Retaining walls are normally built as an appurtenance to other structures: dams, hydroelectric power houses, pump stations, etc. The consequences of failure of a retaining wall are often lower than for flood walls. Also, retaining walls are seldom more than a few hundred feet long; if they are designed conservatively, the added costs are of limited significance. Flood walls, on the other hand, are usually the primary feature of a local protection project. They must be designed for the most economical cross section per unit length of wall, because they often extend for great distances. Added to this need for an economical cross section is the requirement for safety. The consequences of failure for a flood wall are normally very great since it protects valuable property and human life. Thus, the design of retaining and flood walls is a complex process involving safety and economy factors, and design must be executed in a logical, conservative manner based on the function of the wall and the consequences of failure. Design documents should describe the decisions leading to the final degree of conservatism.

2-14. Stability Considerations. An adequate assessment of stability must include a rational assessment of loads and must account for the basic structural behavior, the mechanism of transmitting compressive and shearing loads to the foundation, the reaction of the foundation to such loads, and the secondary effects of the foundation behavior on the structure.

Section IV. Coordination Between Disciplines

2-15. Engineering Team. A fully coordinated team of geotechnical and structural engineers, and hydraulic engineers where applicable, should ensure that all pertinent engineering considerations are properly integrated into the overall design of a structure. Some of the critical aspects of design which require coordination are:

a. Preliminary estimates of geotechnical and hydraulic data, subsurface conditions, and types of structures which are suitable for the foundation.

b. Selection of design parameters, loading conditions, loading effects, potential failure mechanisms, and other related features of the analytical models.

c. Evaluation of the technical and economic feasibility of alternative types of structures.

d. Constructability reviews in accordance with ER 1110-1-803.

e. Refinements of the preliminary structure configuration to reflect the results of detailed site explorations, material availability studies, laboratory testing, and numerical analysis.

f. Modification to the structure configuration during construction due to unexpected variations in the foundation conditions.

Section V. Geotechnical Investigations

2-16. Planning the Investigation.

a. Purpose. The purpose of the geotechnical investigation for wall design is to identify the type and distribution of foundation materials, to identify sources and characteristics of backfill materials, and to determine material parameters for use in design analyses. Specifically, the information obtained will be used to select the foundation type and depth, design the foundation, estimate backfill pressures, locate the ground-water level, estimate settlements, and identify possible excavation problems. For flood walls, foundation underseepage conditions must also be assessed. Detailed information regarding subsurface exploration techniques may be found in EM 1110-1-1804 and EM 1110-2-1907.

b. Review of Existing Information. The first step in an investigational program is to review existing data so that the program can be tailored to confirm and extend the existing knowledge of soil and rock conditions. EM 1110-1-1804 provides a detailed listing of possible data sources; important sources include air photographs, geologic maps, surficial soil maps, and logs from previous borings. In the case of flood walls, study of old topographic maps can provide information on past riverbank or shore geometry and identify likely fill areas.

2-17. Foundation Exploration and Site Characterization.

a. Preliminary Exploration. Where possible, exploration programs should be accomplished in phases, so that information obtained in each phase may be used advantageously in planning later phases. The results of each phase are used to "characterize" the site deposits for analysis and design by developing idealized material profiles and assigning material properties. For long, linear structures like flood walls, geophysical methods such as seismic and resistivity techniques often provide an ability to rapidly define general conditions during the preliminary phase at a modest cost. In alluvial floodplains, air photograph studies can often locate recent channel fillings or other potential problem areas. A moderate number of borings should be obtained at the same time to refine the site characterization and to "calibrate" geophysical findings. Borings should extend deep enough to sample any materials which may affect wall performance; a depth of twice the wall height below the ground surface can be considered a conservative "rule of thumb." For flood walls where underseepage is of concern, a sufficient number of the borings should extend deep enough to establish the thickness of any pervious strata.

b. Detailed Exploration. The purpose of this phase is the development of detailed material profiles and quantification of material parameters. The number of borings should typically be two to five times the number of preliminary borings. No exact spacing is recommended, as the boring layout should consider geologic conditions and the characteristics of the proposed structure. Based on the preliminary site characterization, borings should be situated to confirm the location of significant changes in foundation conditions as well as to confirm the continuity of apparently consistent foundation conditions. At this time, undisturbed samples should be obtained for laboratory testing and/or in situ tests should be performed.

c. Additional Exploration. In some cases, additional exploration phases may be useful to resolve questions arising during detailed design, and/or to provide more detailed information to bidders in the plans and specifications.

2-18. Testing of Foundation Materials.

a. General. Procedures for testing soils are described in EM 1110-2-1906. Procedures for testing rock specimens are described in the Rock Testing Handbook (U. S. Army Engineer Waterways Experiment Station (WES) 1980). Much of the discussion on use of laboratory tests in EM 1110-1-1804 and EM 1110-2-1913 also applies to wall design. For wall design, classification and index tests (water content, Atterberg limits, grain size) should be performed on most or all samples and shear tests should be performed on selected representative undisturbed samples. Where settlement of fine-grained foundation materials is of concern, consolidation tests should also be performed. The strength parameters ϕ and c are not intrinsic material properties but rather are parameters that depend on the applied stresses, the degree of consolidation under those stresses, and the drainage conditions during shear. Consequently, their values must be based on laboratory tests that appropriately model these conditions as expected in the field.

b. Coarse-Grained Materials. Coarse-grained materials such as clean sands and gravels are sufficiently pervious that excess pore pressures do not develop when stress conditions are changed. Their behavior can be modeled for static analyses (earth pressure, sliding, bearing) using parameters from consolidated-drained (S) tests. Failure envelopes plotted in terms of total or effective stresses are the same, and typically exhibit a zero c value and a ϕ value in the range of 25 to 40 degrees. Because of the difficulty of obtaining undisturbed samples of coarse-grained foundation materials, the ϕ value is usually inferred from in situ tests or conservatively assumed based on material type. Where site-specific correlations are desired for important structures, laboratory tests may be performed on samples recompacted to simulate field density.

c. Fine-Grained Materials.

(1) When fine-grained materials such as silts and clays are subjected to stress changes, excess (positive or negative) pore pressures are induced because their low permeability precludes an instantaneous water content change. Undrained (Q or R) tests model such behavior. Shear strength envelopes for undrained tests plotted in terms of total stresses exhibit a non-zero c parameter. However, if plotted in terms of effective stresses, the c parameter is small (zero for all practical purposes) and the friction angle will be essentially equal to that from a drained test. Reasonable estimates of the drained friction angle ϕ' can often be made using correlations with the plasticity index (Figure 2-4).

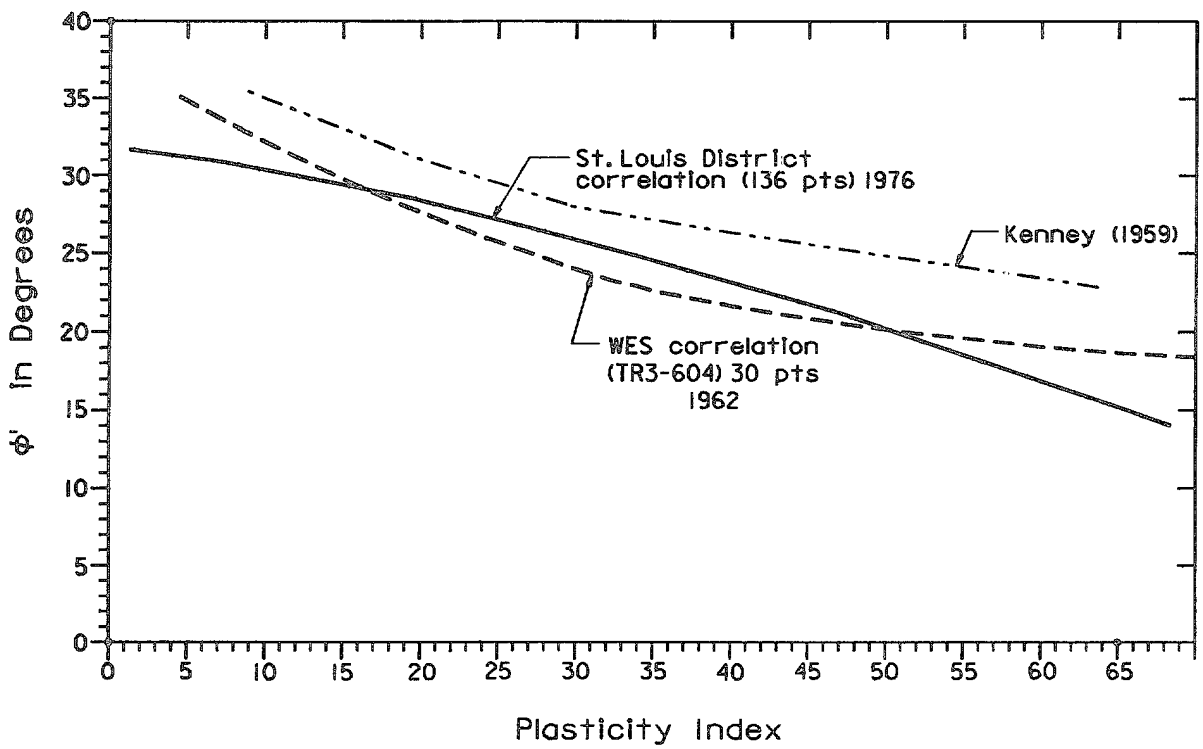


Figure 2-4. Drained friction angle versus plasticity index

(2) At low stress levels, such as near the top of a wall, the undrained strength is greater than the drained strength due to the generation of negative pore pressures which can dissipate with time. Such negative pore pressures allow steep temporary cuts to be made in clay soils. Active earth pressures calculated using undrained parameters are minimum (sometimes negative) values that may be unconservative for design. They should be used, however, to calculate crack depths when checking the case of a water-filled crack.

(3) At high stress levels, such as below the base of a high wall, the undrained strength is lower than the drained strength due to generation of positive pore pressures during shear. Consequently, bearing capacity and sliding analyses of walls on fine-grained foundations should be checked using both drained and undrained strengths.

(4) Certain materials such as clay shales exhibit greatly reduced shear strength once shearing has initiated. For walls founded on such materials, sliding analyses should include a check using residual shear strengths.

2-19. In Situ Testing of Foundation Materials.

a. Advantages. For designs involving coarse-grained foundation materials, undisturbed sampling is usually impractical and in situ testing is the only way to obtain an estimate of material properties other than pure assumption. Even where undisturbed samples can be obtained, the use of in situ methods to supplement conventional tests may provide several advantages: lower costs, testing of a greater volume of material, and testing at the in situ stress state. Although numerous types of in situ tests have been devised, those most currently applicable to wall design are the standard penetration test, the cone penetration test, and the pressuremeter test.

b. Standard Penetration Test. The standard penetration test or SPT (ASTM D-1586) is routinely used to estimate the relative density and friction angle of sands using empirical correlations. To minimize effects of overburden stress, the penetration resistance, or N value, is usually corrected to an effective vertical overburden stress of 1 ton per square foot using an equation of the form:

$$N' = C_N N \qquad [2\text{-}1]$$

where

N' = corrected resistance

C_N = correction factor

N = measured resistance

Table 2-1 and Figure 2-5 summarize the most commonly proposed values for C_N. The drained friction angle ϕ' can be estimated from N' using Figure 2-6. The relative density of normally consolidated sands can be estimated from the correlation obtained by Marcuson and Bieganousky (1977):

$$D_r = 11.7 + 0.76\left[\left|222(N) + 1600 - 53(p'_{vo}) - 50(C_u)^2\right|\right]^{1/2} \qquad [2\text{-}2]$$

where

p'_{vo} = effective overburden pressure in pounds per square inch

C_u = coefficient of uniformity

Correlations have also been proposed between the SPT and the undrained strength of clays. However, these are generally unreliable and should only be used for very preliminary studies and for checking the reasonableness of SPT and lab data.

c. Cone Penetration Test. The cone penetration test, or CPT (ASTM D 3441-79), is widely used in Europe and is gaining considerable acceptance in the United States. The interpretation of the test is described by Robertson and Campanella (1983). For coarse-grained soils, the cone resistance q_c has been empirically correlated with standard penetration resistance (N value). The ratio (q_c/N) is typically in the range of 2 to 6 and is related to median grain size (see Figure 2-7). The undrained strength of fine-grained soils may be estimated by using a modification of bearing capacity theory:

$$s_u = \frac{q_c - p_o}{N_k} \qquad [2\text{-}3]$$

where

p_o = the in situ total overburden pressure

N_k = empirical cone factor typically in the range of 10 to 20

The N_k value should be based on local experience and correlation to laboratory tests. Cone penetration tests also may be used to infer soil classification to supplement physical sampling. Figure 2-8 indicates probable soil type as a function of cone resistance and friction ratio. Cone penetration tests may produce erratic results in gravelly soils.

Table 2-1

SPT Correction to 1 tsf (2 ksf)

Effective Overburden Stress (kips/sq ft)	Correction Factor C_N Seed, Arango, and Chan (1975) Seed	Peck and Bazaraa (1969) P & B	Peck, Hanson, and Thornburn (1974) PH & T
0.20	2.25	2.86	
0.40	1.87	2.22	1.54
0.60	1.65	1.82	1.40
0.80	1.50	1.54	1.31
1.00	1.38	1.33	1.23
1.20	1.28	1.18	1.17
1.40	1.19	1.05	1.12
1.60	1.12	0.99	1.08
1.80	1.06	0.96	1.04
2.00	1.00	0.94	1.00
2.20	0.95	0.92	0.97
2.40	0.90	0.90	0.94
2.60	0.86	0.88	0.91
2.80	0.82	0.86	0.89
3.00	0.78	0.84	0.87
3.20	0.74	0.82	0.84
3.40	0.71	0.81	0.82
3.60	0.68	0.79	0.81
3.80	0.65	0.78	0.79
4.00	0.62	0.76	0.77
4.20	0.60	0.75	0.75
4.40	0.57	0.73	0.74
4.60	0.55	0.72	0.72
4.80	0.52	0.71	0.71
5.00	0.50	0.70	0.70

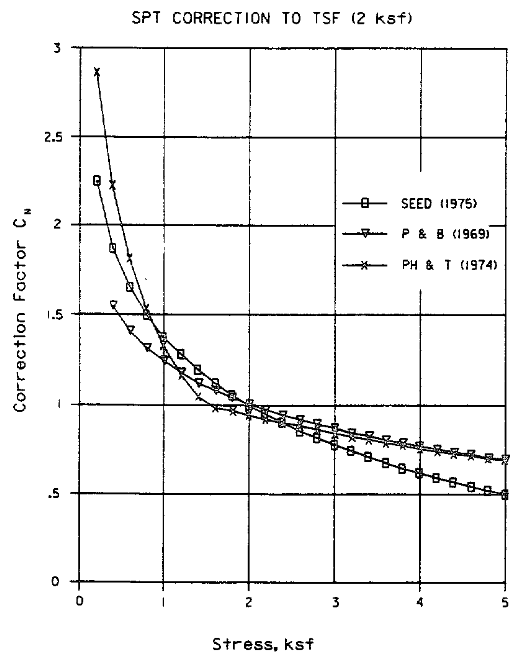


Figure 2-5. SPT correction to 1 tsf (2 ksf)

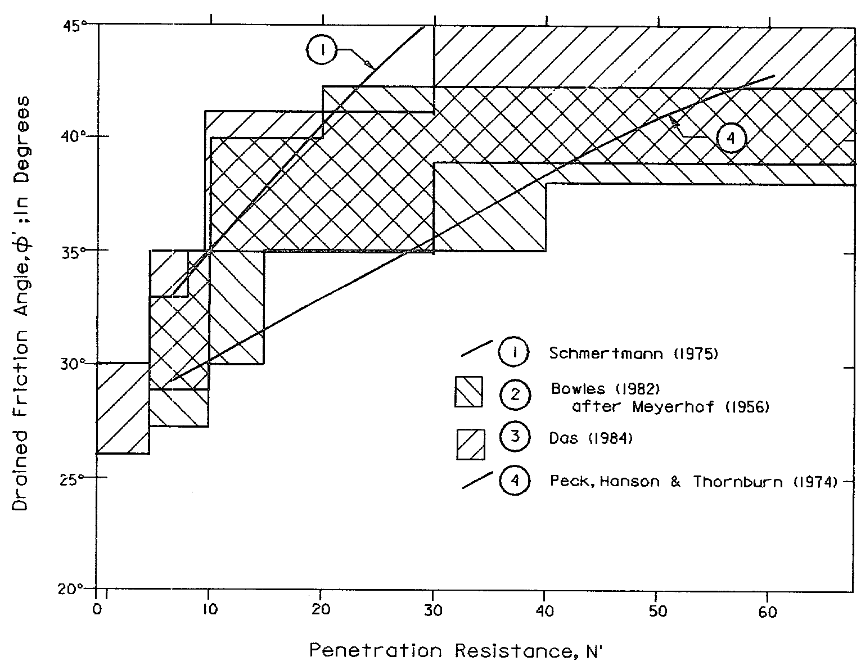


Figure 2-6. ϕ' versus N' for granular materials

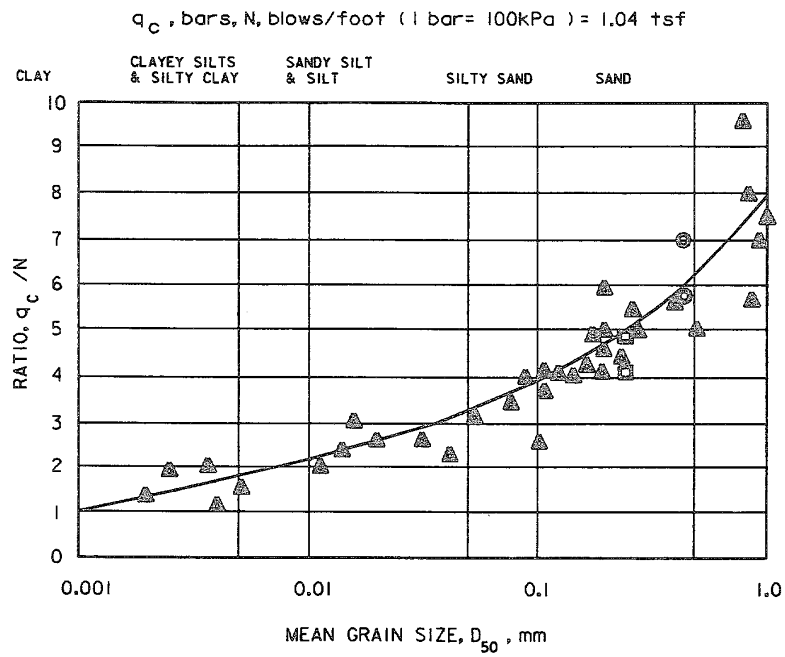


Figure 2-7. q_c/N versus D_{50} (after Robertson and Campanella 1983)

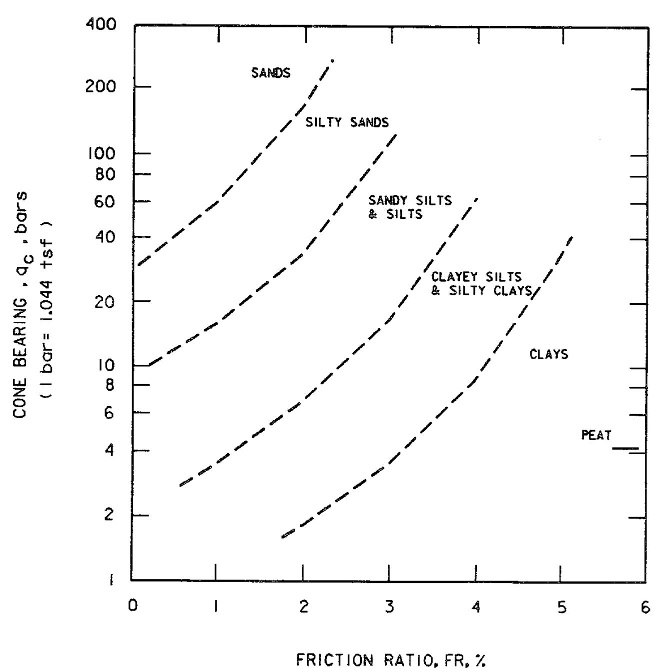


Figure 2-8. Soil classification from cone penetrometer (after Robertson and Campanella 1983)

d. Pressuremeter Test. The pressuremeter test, or PMT, also originated in Europe. Its use and interpretation are discussed by Baguelin, Jezequel, and Shields (1978). Test results are normally used to directly calculate bearing capacity and settlements, but the test can be used to estimate strength parameters. The undrained strength of fine-grained materials is given by:

$$s_u = \frac{p_1 - p'_{ho}}{2K_b} \qquad [2\text{-}4]$$

where

p_1 = limit pressure

p'_{ho} = effective at-rest horizontal pressure

K_b = a coefficient typically in the range of 2.5 to 3.5 for most clays.

Again, correlation with laboratory tests and local experience is recommended.

2-20. Backfill Materials. Selection of backfill materials is discussed in Chapter 6. Every effort should be made to provide clean, free-draining backfill materials. Density and strength parameters should be determined from tests on laboratory-compacted samples over a range of densities consistent with expected specification requirements. Development of a local data base and correlations for the properties of locally obtained backfill materials may significantly reduce the need for testing. Figure 2-9 provides typical values of the friction angle for use in preliminary designs. The soil type codes are taken from the Unified Soil Classification System, shown in Technical Memorandum 3-357, prepared by the U. S. Army Engineer Waterways Experiment Station in 1960. The data for this figure were assembled from a wide variety of design references.

2-21. Design Strength Selection. As soils are heterogeneous (or random) materials, strength tests invariably exhibit scattered results. The guidance contained in EM 1110-2-1902 regarding the selection of design strengths at or below the thirty-third percentile of the test results is also applicable to walls. For small projects, conservative selection of design strengths near the lower bound of plausible values may be more cost-effective than performing additional tests. Where expected values of drained strengths (ϕ values) are estimated from correlations, tables, and/or experience, a design strength of 90 percent of the expected (most likely) value will usually be sufficiently conservative. In the case of rock foundations, the strength of intact rock, the strength and orientation of discontinuities, and the orientation of joints relative to the possible failure modes must all be considered in selecting design strengths.

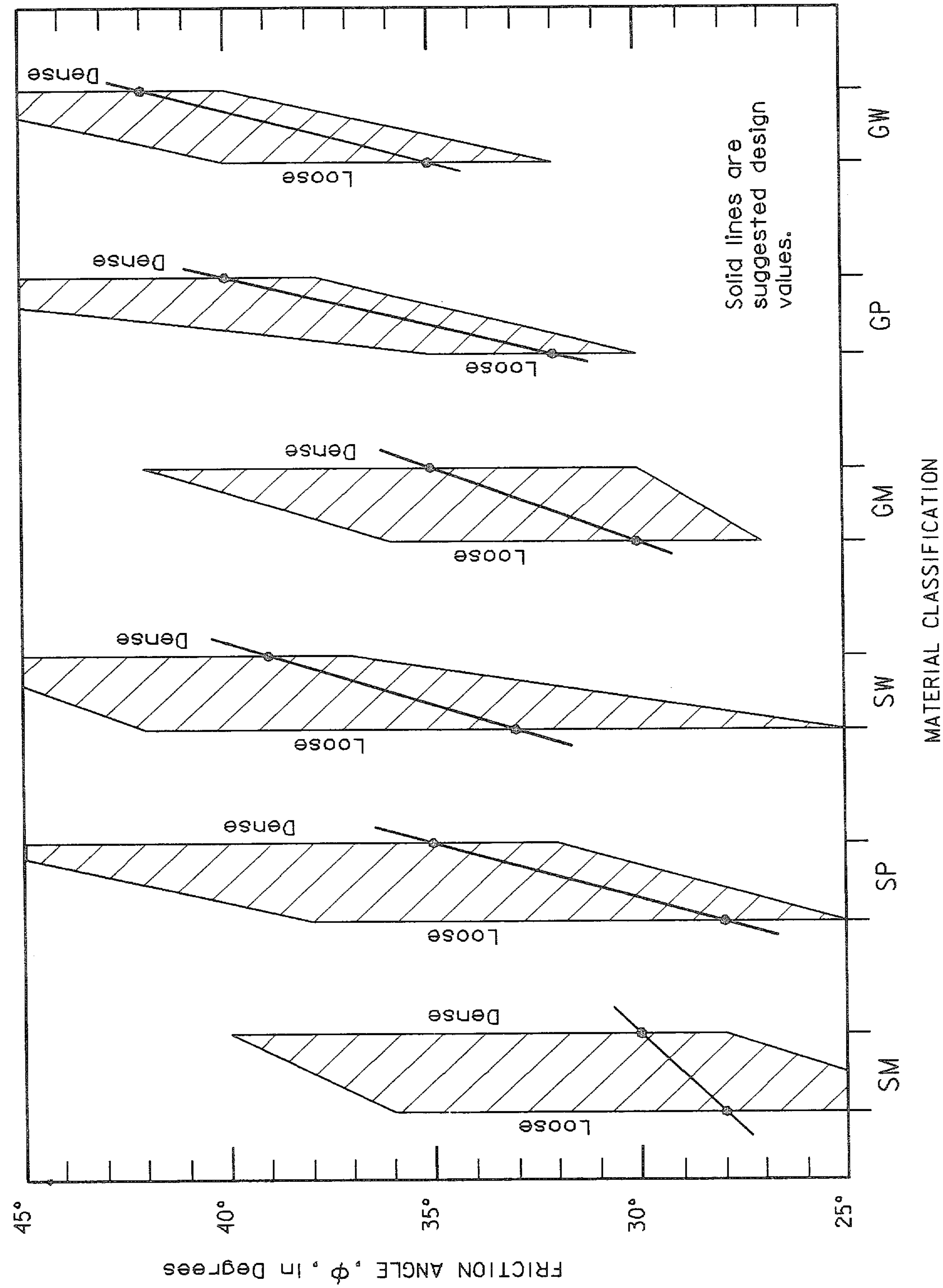


Figure 2-9. Friction angle of granular backfills